GEOTECHNICAL EVALUATION
NRG EL SEGUNDO POWER REDEVELOPMENT
301 VISTA DEL MAR
EL SEGUNDO, CALIFORNIA

PREPARED FOR:

Shaw Stone & Webster Engineering Consultant
9201 East Drycreek Road
Centennial, Colorado 80112

PREPARED BY:

Ninyo & Moore
Geotechnical and Environmental Sciences Consultants
475 Goddard, Suite 200
Irvine, California 92618

April 26, 2007
Project No. 206954002

April 26, 2007
Project No. 206954002

Mr. James Meisenheimer
Shaw Stone & Webster Engineering Consultants
9201 East Drycreek Road
Centennial, Colorado 80112

Subject: Geotechnical Evaluation
NRG El Segundo Power Redevelopment
301 Vista del Mar
El Segundo, California

Dear Mr. Meisenheimer:

In accordance with your authorization, Ninyo & Moore has performed a geotechnical evaluation for the proposed widening of the existing access road and the construction of two 40-foot-diameter, steel condensate tanks south of Plant Units 3 and 4 at the NRG El Segundo Power Plant located at 301 Vista del Mar in El Segundo, California. This evaluation was conducted in general accordance with our proposal dated January 23, 2007. This report presents our findings, conclusions, and recommendations regarding the geotechnical aspect of the access road widening and condensate tank construction.

We appreciate the opportunity to be of service on this project.

Sincerely,
NINYO & MOORE

Original Document Signed By:

Soumitra Guha, Ph.D., G.E.
Principal Engineer

Original Document Signed By:

Carol A. Price, C.E.G.
Principal Geologist

MKM/SG/CAP/emp

Distribution: (3) Addressee

TABLE OF CONTENTS

Page

Tables

Figures

Appendices

1. INTRODUCTION

In accordance with your request and authorization, we have performed a geotechnical evaluation for the proposed access road widening and 40-foot-diameter, steel condensate tanks which are part of the NRG El Segundo Power Redevelopment Project at the existing NRG El Segundo Power Plant in El Segundo, California (Figure 1). The temporary ramp referenced in our proposal dated January 23, 2007 (Ninyo & Moore, 2007a), has since been removed from our scope of work. We previously performed a limited geotechnical evaluation for the proposed redevelopment of Units 1 and 2 at the NRG El Segundo facility (Ninyo & Moore, 2006) and a supplemental evaluation of the stability of the existing slope located along the east side of the redevelopment site for Units 1 and 2 (Ninyo & Moore, 2007b). The purpose of this study was to conduct a geotechnical evaluation in the areas of the proposed access road widening and steel condensate tanks in order to evaluate the subsurface soils and existing pavement conditions and to provide geotechnical recommendations pertaining to the design and construction of these structures. This report presents our geotechnical findings, conclusions, and recommendations regarding this project.

2. SCOPE OF SERVICES

Geotechnical services during this evaluation were provided in general accordance with our proposal dated January 23, 2007 (Ninyo & Moore, 2007a), and included the following:

- Project coordination and review of readily available background materials, including geologic and topographic maps, published literature, stereoscopic aerial photographs, in-house information, and Cone Penetration Test (CPT) data obtained from a previous subsurface evaluation performed by Ninyo & Moore for the Units 1 and 2 Redevelopment Project (Ninyo & Moore, 2006).

- Performance of a site reconnaissance to mark the proposed boring locations and to coordinate with Underground Service Alert (USA) for utility clearance.

- Subsurface exploration consisting of the drilling, sampling, and logging of four small-diameter, hollow-stem auger borings; two borings were performed in the area of the proposed access road widening to a depth of approximately 12½ feet below the paved surface and two borings were performed in the areas of the proposed condensate steel tanks to a depth of approximately 31½ feet below the existing grade.

- Laboratory testing of representative soil samples to evaluate in-place moisture content and dry density, percent of particles finer than the No. 200 sieve, direct shear strength, R-value, sand equivalent, and corrosivity.
- Data compilation and geotechnical analysis of the field and laboratory data.
- Preparation of this report presenting our geotechnical findings, conclusions, and recommendations regarding the project.

3. SITE DESCRIPTION

The site for the proposed project is located within the existing NRG El Segundo Power Plant at 301 Vista Del Mar in El Segundo, California (Figure 1). The access road is a two-lane roadway that extends from Vista del Mar to the northwest for approximately 450 feet and descends approximately 70 feet to the south of Units 3 and 4 (i.e., from an elevation of approximately 91 feet above mean low level water [MLLW] to approximately 21 feet above MLLW). Units 3 and 4 are situated at the base of the access road on relatively level terrain near the southern end of El Segundo Beach, at an elevation of approximately 19½ feet above MLLW. The site for the proposed tanks is currently paved with asphalt concrete.

4. PROPOSED CONSTRUCTION

We understand that the proposed improvements will consist of the widening of the existing access road and the construction of two steel condensate tanks. The access road widening will extend from approximately 60 feet south of the existing helipad drive path to approximately 200 feet to the north of the drive path. In order to accommodate the roadway widening, the existing slope along the west side of the access road will be cut. The proposed steel condensate tanks will be located in an area to the south of Units 3 and 4. We understand that the 40-foot-diameter tanks will be approximately 38 feet tall and will be supported on ring beams.

5. SUBSURFACE EVALUATION AND LABORATORY TESTING

Our subsurface exploration at the subject site was performed on February 14, 2007, and consisted of the drilling, logging, and sampling of four small-diameter borings. The approximate locations of the exploratory borings were selected by Shaw, Stone & Webster and are shown on Figure 2. Two borings were drilled to a depth of approximately 12½ feet below the pavement surface in the area of the access road widening, and two borings were drilled to a depth of approximately 31½ feet below the pavement surface in the areas of the proposed condensate tanks. The borings were logged and sampled by a representative from our firm. Bulk and relatively undisturbed soil samples were obtained at selected depths for laboratory testing. The logs of the exploratory borings are presented in Appendix A.

Laboratory testing of representative soil samples was performed to evaluate in-situ moisture content and dry density, percent of particles finer than the No. 200 sieve, direct shear strength, R-value, sand equivalent, and corrosivity. The results of our in-situ moisture content and dry density evaluation are presented on the boring logs in Appendix A. The remaining laboratory testing results are presented in Appendix B.

6. GEOLOGY AND SUBSURFACE CONDITIONS

6.1. Regional Geology

The site for the proposed improvements is located within the Los Angeles Basin, which is bounded on the north by the Transverse Ranges geomorphic province (Norris and Webb, 1990). The Los Angeles Basin has been divided into four blocks, which are generally separated by prominent fault systems: the northwestern block, the southwestern block, the central block, and the northeastern block. The project area is located within the southwestern block, which is bounded on the east by the onshore segment of the Newport-Inglewood fault zone. The southwestern block includes anticlinal and synclinal structural features within the basement rocks that are overlain by younger sedimentary rocks and alluvium.

The Los Angeles Basin is traversed by several major active faults. The Palos Verdes and Newport-Inglewood fault zones are major active faults within the southwestern block of the Los Angeles Basin. Our review of geologic literature indicates that a segment of the Palos

Verdes fault is located about 3 miles southwest of the site. The on-shore segment of the Newport-Inglewood fault is located approximately 6 miles northeast of the site.

6.2. Site Geology

Based on our review of stereoscopic aerial photographs and pertinent geologic maps, the site is underlain by Holocene-age alluvial and dune deposits consisting of silty sand and sand. The subsurface materials encountered in the tank borings (i.e., T-1 and T-2) were older alluvium (underlying the asphalt concrete pavement section) consisting of generally medium dense to very dense, poorly graded sand and silty sand to the explored depth of approximately 31½ feet. The subsurface materials encountered in the roadway borings (i.e., RW-1 and RW-2) were eolian deposits (underlying the asphalt concrete pavement section) consisting of loose to medium dense, poorly graded sand to the explored depth of approximately 12½ feet. Detailed descriptions are presented on the boring logs in Appendix A.

7. GROUNDWATER

Groundwater was encountered in borings T-1 and T-2 at a depth of approximately 13½ feet below the ground surface. Groundwater was not encountered in borings RW-1 and RW-2 within the explored depth. Fluctuations in groundwater levels may occur due to tidal fluctuations, variations in precipitation, ground surface topography, subsurface stratification, local irrigation, and other factors which may not have been evident at the time of our field evaluation.

8. FAULTING AND SEISMICITY

Based on our review of the referenced geologic maps and stereoscopic aerial photographs, the ground surface in the vicinity of the subject site is not mapped as being transected by any known active or potentially active fault; therefore, the potential for surface fault rupture is considered to be low. The site is not located within a State of California Earthquake Fault Zone (Alquist-Priolo Special Studies Zone, Hart and Bryant, 1997). However, the subject site is located in a seismically active area, as is the majority of southern California, and the potential for strong ground motion at the site is considered significant. The nearest known active fault is the Palos Verdes fault located approximately 3 miles southwest of the site.

Table 1 lists selected principal known active faults that may affect the subject site, the maximum moment magnitude (M_{max}) as published for the California Geological Survey (CGS) by Cao, et al. (2003), and the type of fault as defined in Table 16A-U of the California Building Code (CBC, 2001). The approximate fault to site distances were calculated using the computer program FRISKSP (Blake, 2001a).

Table 1 – Principal Active Faults

Fault	Approximate Fault to Site Distance in miles (km)	Maximum Moment Magnitude[1] (M_{max})	Fault Type[2]
Palos Verdes	3.2 (5.2)	7.3	B
Newport-Inglewood (Los Angeles Basin)	6.2 (10.0)	7.1	B
Santa Monica	9.3 (14.9)	6.6	B
Malibu Coast	10.4 (16.8)	6.7	B
Hollywood	12.1 (19.5)	6.4	B
Puente Hills Blind Thrust	12.6 (20.3)	7.1	B
Northridge	16.8 (27.0)	7.0	B
Verdugo	20.4 (32.8)	6.9	B
Sierra Madre	24.8 (39.9)	7.2	B
San Andreas – 1857 Rupture	47.7 (76.8)	7.4	A

Notes:
[1] Cao et al. 2003.
[2] CBC, 2001; Cao et al., 2003.

The principal seismic hazards at the subject site are surface ground rupture, ground shaking, seismically induced liquefaction, and various manifestations of liquefaction-related hazards (e.g., dynamic settlements and lateral spreading). A brief description of these hazards and the potential for their occurrences on site are discussed in the following sections.

8.1. Ground Rupture

The probability of damage from surface ground rupture is low due to the lack of known active faults underlying the subject site or its vicinity. Surface ground cracking related to shaking from distant events is not considered a significant hazard, although it is a possibility.

8.2. Ground Motion

Our evaluation of the ground shaking hazard included review of a probabilistic seismic hazard assessment that consisted of statewide estimates of peak horizontal ground accelerations conducted for California (Peterson, et al., 1996). In addition, for the purposes of evaluating seismically induced geotechnical hazards at the site, a site-specific probabilistic seismic hazard analysis was performed to evaluate anticipated peak ground accelerations (PGAs) using the computer program FRISKSP (Blake, 2001a). A probabilistic analysis incorporates uncertainties in time, recurrence intervals, size, and location (along faults) of hypothetical earthquakes. This method thus accounts for likelihood (rather than certainty) of occurrence and provides levels of ground acceleration that might be more reasonably hypothesized for a finite exposure period. FRISKSP calculates the probability of occurrence of various ground accelerations at a site over a period of time and the probability of exceeding expected ground accelerations within the lifetime of the proposed structures from the significant earthquakes within a specific radius of search. For the present case, a search radius of 62 miles (i.e., 100 kilometers) was selected. The earthquake magnitudes used in this program are based on the current CGS fault model.

The published guidelines of CGS (2004) define a PGA with a 10 percent probability of exceedance in 50 years as the Design Basis Earthquake (PGA_{DBE}) ground motion, and this value is typically used for residential, commercial, and industrial structures. The PGA with a 10 percent probability of exceedance in 100 years is defined as the Upper Bound Earthquake (PGA_{UBE}) ground motion and is used for public schools, hospitals, and other essential facilities in California. The statistical return periods for the PGA_{DBE} and PGA_{UBE} are approximately 475 and 949 years, respectively.

In evaluating the seismic hazards associated with the subject site, we have considered a PGA that has a 10 percent probability of being exceeded in 50 years (i.e., PGA_{DBE}) and used an attenuation relation proposed by Boore, et al. (1997), for soil Type D (with an average shear wave velocity of 820 feet or 250 meters per second). The PGA_{DBE} for the site was calculated as 0.39g when weighted to an earthquake magnitude of 7.5. The PGA_{DBE} increases to 0.48g when no magnitude weighting factor is considered in probabilistic seismic hazard analysis.

These estimates of ground motion do not include near-source factors that may be applicable in the design of structures on site.

8.3. Liquefaction

Liquefaction is the phenomenon in which loosely deposited, saturated, granular soils (located below the water table) with clay contents (particles less than 0.005 millimeters [mm]) of less than 15 percent, liquid limit of less than 35 percent, and natural moisture content greater than 90 percent of the liquid limit undergo rapid loss of shear strength due to development of excess pore pressure during strong earthquake-induced ground shaking. Ground shaking of sufficient duration results in the loss of grain-to-grain contact due to a rapid rise in pore water pressure, and it eventually causes the soil to behave as a fluid for a short period of time. Liquefaction is known generally to occur in saturated or near-saturated cohesionless soils at depths shallower than 50 feet below the ground surface. Factors known to influence liquefaction potential include composition and thickness of soil layers, grain size, relative density, groundwater level, degree of saturation, and both intensity and duration of ground shaking.

Based on our review of the State of California Seismic Hazards Zones map (California Division of Mines and Geology [CDMG], 1999), the subject site is not located in a mapped liquefaction hazard zone but is located approximately 300 feet from an area mapped as being susceptible to liquefaction during a seismic event. A preliminary liquefaction evaluation of subsurface soils was performed (Ninyo & Moore, 2006) for Units 1 and 2 located on the north side of the site. A historic high groundwater level at a depth of 5 feet below the existing grade was considered in that evaluation. The liquefaction analysis was based on the NCEER procedure (Youd and Idriss, 1997) developed from the methods originally recommended by Seed and Idriss (1982) using the computer program LIQUEFY2 (Blake, 2001b). A magnitude-weighted PGA_{DBE} of 0.39g was used in the analysis for an earthquake magnitude of 7.5. Results of our liquefaction evaluation for Units 1 and 2 indicated that some of the granular soil layers located below the historic high groundwater level might liquefy during the design seismic event to a depth of approximately 15 feet below the ground surface.

Comparing the nature and density of subsurface soils encountered in our recent borings for the condensate tanks to that observed at Units 1 and 2, we anticipate that liquefaction would likely occur in subsurface soils underlying the tanks to depths of approximately 15 to 20 feet below the existing grade under historic high groundwater condition.

8.4. Dynamic Settlement of Saturated Soils

The phenomenon of soil liquefaction may result in several hazards, including liquefaction-induced settlement. In order to estimate the amount of post-earthquake settlement, the method proposed by Tokimatsu and Seed (1987) is generally used in which the seismically induced cyclic stress ratios and corrected blow counts (N-values) are correlated to the volumetric strain of the soil. The amount of soil settlement during a strong seismic event depends on the thickness of the liquefiable layers and the density and/or consistency of the soils.

Based on our recent evaluation for Units 1 and 2 (Ninyo & Moore, 2006) and our experience with liquefiable soils in the general vicinity of the project site, a post-earthquake dynamic ground settlement of up to approximately 2 inches may occur in relatively saturated soils located below the historic high groundwater level at the tank site. Based on the guidelines presented in CDMG Special Publication 117 (1997), we estimate that differential settlement on the order of 1 inch may occur over a horizontal distance of 20 feet. The dynamic settlement magnitudes may vary across the site; the estimates presented here should be considered preliminary.

8.5. Ground Subsidence

The potential for ground subsidence, sand boils, and/or seismically induced bearing failure is considered to be moderate if the tanks are to be constructed at the present grade. In the event the site grade is raised, the potential for ground subsidence will be reduced.

8.6. Lateral Spreading

Lateral spreading of the ground surface during an earthquake usually takes place along weak shear zones that have formed within a liquefiable soil layer. Lateral spread has generally been observed to take place in the direction of a free-face (i.e., retaining wall, slope, channel) but has also been observed to a lesser extent on ground surfaces with gentle slopes. An empirical model developed by Bartlett and Youd (1995, revised 1999) is typically used to predict the amount of horizontal ground displacement within a site. For sites located in proximity to a free-face, the amount of lateral ground displacement is strongly correlated with the distance of the site from the free-face. Other factors such as earthquake magnitude, distance from the earthquake epicenter, thickness of the liquefiable layers, and the fines content and particle sizes of the liquefiable layers also affect the amount of lateral ground displacement. Based on the relative density of the potentially liquefiable soil layers, the site is not considered susceptible to seismically induced lateral spread.

8. CONCLUSIONS

Based on the results of our geotechnical evaluation, it is our opinion that construction of the proposed improvements is feasible from a geotechnical perspective if the recommendations presented in this report are incorporated into the design and construction of the project. In general, the following conclusions were made:

- The site is underlain by fill, older alluvium, and eolian soils generally consisting of alternating layers of medium dense to very dense sand, and silty sand to the explored depths.

- Near-surface alluvial soils encountered in our exploratory borings within the tank site are not considered suitable for supporting the proposed tanks and may be subject to settlement under applied loads. To mitigate the potential for future settlement, these soils should be removed and replaced as compacted fill. Remedial grading recommendations are presented in Section 9.1 of this report.

- Groundwater was encountered in our tank borings (i.e., T-1 and T-2) at a depth of approximately 13½ feet below the ground surface and was not encountered in borings RW-1 and RW-2. Groundwater should be anticipated and planned for by the contractor during construction of deeper foundation elements, if any, for the proposed tanks.

- The fill, alluvial, and eolian soils should be generally excavatable with earthmoving equipment in good working condition

- We estimate a Design Basis peak ground acceleration (PGA_{DBE}) of 0.39g for an earthquake magnitude of 7.5 at the subject site that has a 10 percent probability of being exceeded in 50 years

- The subsurface soils are generally susceptible to liquefaction during the design seismic event. Our analysis indicates that granular soil layers located below the historic high groundwater level could liquefy during the design seismic event up to a depth of about 20 feet below the existing grade.

- A post-earthquake dynamic ground settlement of up to approximately 2 inches may occur in relatively saturated soils located below the historic high groundwater. We estimate that differential settlement on the order of 1 inch may occur over a horizontal distance of 20 feet.

- The potential for ground subsidence, sand boils, and/or seismically induced bearing failure is considered to be relatively moderate. In the event the site grade for the proposed tanks is raised, the potential for ground subsidence will be reduced.

- Liquefaction-induced lateral spread is not expected at the project site.

- The site is not located within a State of California Earthquake Fault Zone (Alquist-Priolo Special Studies Zone). Based on our review of published geologic maps and aerial photographs, no known active or potentially active faults underlie the site. The potential for surface fault rupture at the site is considered to be low.

9. RECOMMENDATIONS

The following recommendations are provided for the design and construction of the proposed improvements. These recommendations are based on our evaluation of the site geotechnical conditions and our understanding of the planned construction, including anticipated tank foundation loads. The proposed site improvements should be constructed in accordance with the requirements of applicable governing agencies.

9.1. Earthwork

Based on our understanding of the project, earthwork is anticipated to consist of removal and recompaction of existing fill and near-surface alluvial and eolian soils, removal of the existing slope in the area of the road widening, and trenching for utility lines. We recommend that the site grading be performed in accordance with the requirements of the applicable governing agency and the following recommendations.

9.1.1. Pre-Construction Conference

We recommend that a pre-construction conference be held in order to discuss the grading recommendations presented in this report. The owner and/or their representative, the governing agencies' representatives, the civil engineer, Ninyo & Moore, and the contractor should be in attendance to discuss the work plan, project schedule, and earthwork requirements.

9.1.2. Site Preparation

Prior to excavation of near-surface soils and placement of fill, the project site should be cleared of existing structures, pavements, abandoned utilities (if present), and stripped of rubble, debris, vegetation, and any loose, wet, or otherwise unstable soils, as well as surface soils containing organic material. Obstructions that extend below the finished grade, if any, should be removed and the resulting holes filled with compacted soil. Materials generated from the clearing operations should be removed from the site and disposed of at a legal dumpsite away from the project area.

9.1.3. Excavation Characteristics

Our evaluation of the excavation characteristics of the on-site materials at the subject site is based on the results of our exploratory borings and our experience with similar materials. In our opinion, the on-site fill, alluvial, and eolian soils should be generally excavatable with heavy-duty earthmoving equipment in good working condition. Gravel-size or larger materials may be encountered during site excavation and should be considered in construction planning.

9.1.4. Treatment of Near-Surface Soils

The near-surface alluvial soils encountered in borings placed at the proposed condensate tank site are not considered suitable for structural foundation support. We recommend that the alluvial soils be removed to a depth of 3 feet below the bottom of the planned lowest foundation bottom elevation and replaced with generally granular, compacted, structural fill with a very low to low expansion potential (i.e., an expansion index [EI] of less than 50 as evaluated in accordance with Uniform Building Code [UBC] Standard 18-2 [International Conference of Building Officials (ICBO), 1997]). In areas of proposed roadway widening and exterior flatwork, the near-surface fill, alluvium, or eolian soils should be removed to a depth of approximately 12 inches below the pavement or flatwork subgrade. The actual depths of overexcavation should be evaluated by our representative based on the materials exposed at the time of construction. The limits of overexcavation should extend laterally beyond the improvements to a distance equal to the depth of overexcavation. Any unsuitable materials, such as organic matter or oversized material, should be selectively removed and disposed of offsite.

9.1.5. Temporary Excavations

We recommend that trenches and excavations be designed and constructed in accordance with Occupational Safety and Health Administration (OSHA) regulations. These regulations provide trench sloping and shoring design parameters for trenches up to 20 feet deep based on the soil types encountered. Trenches over 20 feet deep should be designed by the contractor’s engineer based on site-specific geotechnical analyses. For planning purposes, we recommend that fill, older alluvium, and eolian soils be considered as OSHA soil type C.

Temporary excavations should be constructed in accordance with OSHA recommendations. For trench or other excavations, OSHA requirements regarding personnel safety should be met by using appropriate shoring (including trench boxes) or by laying back the slopes no steeper than 1.5:1 (horizontal to vertical) in fill and alluvium. Temporary excavations that encounter seepage may need shoring or may be stabilized by placing sandbags or gravel along the base of the seepage zone. Excavations encountering seepage should be evaluated on a case-by-case basis. On-site safety of personnel is the responsibility of the contractor. Recommendations for temporary shoring can be provided, if requested.

9.1.6. Fill Material

In general, the on-site soils are considered suitable for reuse as fill. On-site fill soils should be free of trash, debris, roots, vegetation, or deleterious materials. Fill should generally be free of rocks or hard lumps of material greater than approximately 4 inches in diameter. Rocks or hard lumps larger than about 4 inches in diameter should be broken into smaller pieces or should be removed from the site. Imported materials, if required, should consist of clean, granular material with a very low to low expansion potential, corresponding to an expansion index (EI) of 50 or less as evaluated by UBC (ICBO, 1997) Standard 18-2. Import materials should also be non-corrosive in accordance with the Caltrans (2003) corrosion guidelines. Import material should be submitted to the project geotechnical consultant for review prior to importing to the site.

9.1.7. Fill Placement and Compaction

Prior to placement of compacted fill, the contractor should request an evaluation of the exposed ground surface by Ninyo & Moore. Unless otherwise recommended, the exposed ground surface should then be scarified to a depth of approximately 12 inches and watered or dried, as needed, to achieve moisture contents generally above the optimum moisture content. The scarified materials should then be compacted to a relative compaction of 95 percent as evaluated in accordance with American Society for Testing and Materials (ASTM) test method D 1557. The evaluation of compaction by the geotechnical consultant should not be considered to preclude any requirements for observation or approval by governing agencies. It is the contractor's responsibility to notify the geotechnical consultant and the appropriate governing agency when the project area is ready for observation and to provide reasonable time for that review.

Fill materials should be moisture conditioned to generally above the laboratory optimum moisture content prior to placement. The optimum moisture content will vary with material type and other factors. Moisture conditioning of fill soils should be generally consistent within the soil mass.

Prior to placement of additional compacted fill material following a delay in the grading operations, the exposed surface of previously compacted fill should be prepared to receive fill. Preparation may include scarification, moisture conditioning, and recompaction.

Compacted fill should be placed in horizontal lifts of approximately 8 inches in loose thickness. Prior to compaction, each lift should be watered or dried as needed to achieve a moisture content generally above the laboratory optimum, mixed, and then compacted by mechanical methods, using sheepsfoot rollers, multiple-wheel pneumatic-tired rollers, or other appropriate compacting rollers, to a relative compaction of 95 percent as evaluated by ASTM D 1557. Successive lifts should be treated in a like manner until the desired finished grades are achieved.

9.1.8. Pipe Bedding and Modulus of Soil Reaction

It is our recommendation that the new pipelines, where constructed in open excavations, be supported on 6 or more inches of granular bedding material. Granular pipe bedding should be provided to distribute vertical loads around the pipe. Bedding material and compaction requirements should be in accordance with this report or in accordance with specification and placement requirements by the pipe supplier. Pipe bedding should have a Sand Equivalent (SE) of 30 or greater and be placed around the sides and the crown of the pipe. In addition, the pipe bedding material should extend 1 foot or more above the crown of the pipe. Bedding material and compaction requirements should be in accordance with the recommendations of this report, the project specifications, and applicable requirements of the appropriate governing agency.

The modulus of soil reaction is used to characterize the stiffness of soil backfill placed at the sides of buried flexible pipes for the purpose of evaluating deflection caused by the weight of the backfill over the pipe (Hartley and Duncan, 1987). A soil reaction modulus of 1,000 pounds per square inch (psi) may be used for an excavation depth of up to about 5 feet when backfilled with granular soil compacted to a relative compaction of 90 percent as evaluated by the ASTM D 1557. A soil reaction modulus of 1,300 psi may be used for trenches deeper than 5 feet.

9.1.9. Trench Backfill

Based on our subsurface evaluation, the on-site soils should be generally suitable for re-use as trench backfill, provided they are free of organic material, clay lumps, debris, and rocks greater than approximately 4 inches in diameter. We recommend that trench back-fill materials be in conformance with the “Greenbook” (Standard Specifications for Public Works Construction) specifications for structure backfill. Fill should be mois-ture-conditioned to generally above the laboratory optimum. Trench backfill should be compacted to a relative compaction of 90 percent as evaluated by the latest edition of ASTM D 1557 except for the upper 12 inches of the backfill which should be com-pacted to a relative compaction of 95 percent as evaluated by the latest edition of ASTM D 1557. Lift thickness for backfill will depend on the type of compaction equipment utilized, but fill should generally be placed in lifts not exceeding 8 inches in loose thickness. Special care should be exercised to avoid damaging the pipe during compaction of the backfill.

9.2. Seismic Design Considerations

Design of the proposed improvements should comply with design for structures located in Seismic Zone 4 and should be designed in accordance with applicable jurisdictions, building codes, and the standard practices of the Structural Engineers Association of California. A soil profile factor of S_D may be utilized in the CBC (CBSC, 2001) seismic design. Addi-tional CBC seismic design parameters are provided in Table 2.

Table 2 – 2001 California Building Code Seismic Parameters

2001 CBC Seismic Design Factor	Value
Seismic Zone Factor, Z	0.4
Seismic Source Type*	B
Near Source Factor, N_a	1.0
Near Source Factor, N_v	1.2
Soil Profile Type	S_D
Seismic Coefficient, C_a	0.44
Seismic Coefficient, C_v	0.77

* Faults are designated as Type A, B or C, depending on maximum moment magnitude and slip rates (Table 16A-U of CBC, 2001).

9.3. Foundations

Foundation recommendations presented in the following sections are for shallow, spread footings bearing on engineered fill compacted in accordance with recommendations presented in Section 9.1 of this report. Foundations should be designed in accordance with structural considerations and the following recommendations. In addition, requirements of the governing jurisdictions, practices of the Structural Engineers Association of California, and applicable building codes should be considered in the design of structures.

9.3.1. Shallow Footings

Shallow, spread or continuous footings founded in compacted fill may be designed using an allowable bearing capacity of 2,500 pounds per square foot (psf). Spread footings should be founded 24 inches below the lowest adjacent grade. Continuous and isolated footings should have a width of 24 inches. The allowable bearing capacity may be increased by 300 psf for every foot of increase in width or depth up to a value of 4,000 psf. These allowable bearing capacities may be increased by one-third when considering loads of short duration, such as wind or seismic forces. The spread footings should be reinforced in accordance with the recommendations of the project structural engineer.

9.3.2. Lateral Resistance

For resistance of footings to lateral loads, we recommend an allowable passive pressure of 350 psf per foot of depth be used with a value of up to 3,500 psf. This value assumes that the ground is horizontal for a distance of 10 feet, or three times the height generating the passive pressure, whichever is greater. We recommend that the upper 1 foot of soil not protected by pavement or a concrete slab be neglected when calculating passive resistance.

For frictional resistance to lateral loads, we recommend a coefficient of friction of 0.40 be used between soil and concrete. The allowable lateral resistance can be taken as the sum of the frictional resistance and passive resistance provided the passive resistance does not exceed one-half of the total allowable resistance. The passive resistance values may be increased by one-third when considering loads of short duration such as wind or seismic forces.

9.3.3. Static Settlement

We estimate that the proposed tanks, designed and constructed as recommended herein, will undergo total settlement on the order of 1 inch. Differential settlement on the order of ½ inch over a horizontal span of 40 feet should be anticipated.

9.4. Pavement Design Considerations

Laboratory testing performed on a sample of representative near-surface soil yielded an R-value 73. We understand that pavement design will be performed by others. We recommend that a design R-value of 60 be considered in pavement section evaluation for the project.

Subgrade soils in areas to be paved should be prepared as recommended in Section 9.1 of this report. Prior to the placement of full-depth asphalt concrete, the upper 12 inches of the subgrade soils should be overexcavated, moisture-conditioned, and compacted to a relative compaction of 95 percent as evaluated by ASTM D 1557. Prior to placement of aggregate base materials, the upper 12 inches of subgrade soils should also be overexcavated, moisture conditioned, and compacted to a relative compaction of 95 percent as evaluated by ASTM D 1557. Aggregate base material should conform to the latest specifications in Section 200-2.0 for crushed aggregate base or Section 200-2.4 for crushed miscellaneous base of the “Greenbook”, and should be compacted to a relative compaction of 95 percent as evaluated by ASTM D 1557.

9.5. Corrosion

The corrosion potential of the near-surface site soils was evaluated using the results of two representative samples obtained from our exploratory borings. Laboratory testing was performed to evaluate pH, minimum electrical resistivity, soluble sulfate, and chloride contents. The pH and electrical resistivity tests were performed in accordance with California Test (CT) 643, and the sulfate and chloride content tests were performed in accordance with CT 417 and 422, respectively. The laboratory test results are presented in Appendix B.

The results of the corrosivity testing indicated electrical resistivity ranging from approximately 6,100 to 6,700 ohm-centimeters, soil pH of 7.5, chloride contents varying between 90 and 115 parts per million (ppm), and sulfate contents ranging from approximately 0.009 to 0.0125 percent (i.e., 90 to 125 ppm). Based on the Caltrans (2003) criteria, the project site would not be classified as corrosive, which is defined as a site having soils with more than 500 ppm of chlorides, more than 0.2 percent sulfates, or a pH less than 5.5.

9.6. Concrete Placement

Concrete in contact with soil or water that contains high concentrations of soluble sulfates can be subject to chemical and/or physical deterioration. Based on the CBC criteria (ICBO, 2001), the potential for sulfate attack is negligible for water-soluble sulfate contents in soil ranging from 0.00 to 0.10 percent by weight (i.e., 0 to 1,000 ppm). The soil samples tested for this evaluation indicate water-soluble sulfate contents ranging from approximately 0.009 to 0.0125 percent by weight (i.e., 90 to 125 ppm). Accordingly, the on-site soils are considered to have a negligible potential for sulfate attack.

9.7. Drainage

Positive surface drainage away from the foundations and berms is imperative for satisfactory site performance. Positive drainage should be provided and maintained to transport surface water away from improvements and off the site. Runoff should then be transported by the use of swales or pipes into a collective drainage system. Surface waters should not be allowed to pond adjacent to footings.

10. CONSTRUCTION OBSERVATION

The recommendations provided in this report are based on our understanding of the proposed project and on our evaluation of the data collected based on subsurface conditions disclosed by widely spaced exploratory borings. It is imperative that the geotechnical consultant checks the interpolated subsurface conditions during construction.

During construction, we recommend that the duties of the geotechnical consultant include, but not be limited to, the following:

- Observing clearing, grubbing, and removals.
- Observing excavation, placement, and compaction of fill.
- Evaluating imported materials prior to their use as fill (if used).
- Performing field tests to evaluate fill compaction.
- Observing foundation excavations for bearing materials and cleaning prior to placement of reinforcing steel or concrete.
- Performing material testing services, including concrete compressive strength and steel tensile strength tests and inspections.

11. LIMITATIONS

The field evaluation and geotechnical analyses presented in this geotechnical report have been conducted in general accordance with current practice and the standard of care exercised by geotechnical consultants performing similar tasks in the project area. No warranty, expressed or implied, is made regarding the conclusions, recommendations, and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be encountered during construction. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation will be performed upon request.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Ninyo & Moore should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

Our conclusions, recommendations, and opinions are based on an analysis of the observed site conditions. If geotechnical conditions different from those described in this report are encountered, our office should be notified and additional recommendations, if warranted, will be provided upon request. It should be understood that the conditions of a site can change with time as a result of natural processes or the activities of man at the subject site or nearby sites. In addition, changes to the applicable laws, regulations, codes, and standards of practice may occur due to government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which Ninyo & Moore has no control.

This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

12. SELECTED REFERENCES

American Concrete Institute (ACI), 1997, ACI Manual of Concrete Practice.

Bartlett, S.F., and Yond, T.L., 1995, Empirical Prediction of Liquefaction-Induced Lateral Spread Journal of Geotechnical Engineering, ASCE, Vol. 121, No. 4, 316-329, dated April.

Blake, T.F., 2001a, FRISKSP (Version 4.00) A Computer Program for the Probabilistic Estimation of Peak Acceleration and Uniform Hazard Spectra Using 3-D Faults as Earthquake Sources.

Blake, T.F., 2001b, LIQUEFY2 (Version 1.50), A Computer Program for the Empirical Prediction of Earthquake-Induced Liquefaction Potential.

Brinderson, 2006, NRG – El Segundo Proposed Site Plan, Scale 1 inch = 80 feet, dated October 9.

California Building Standards Commission, 2001, California Building Code (CBC), Title 24, Part 2, Volumes 1 and 2.

California Department of Conservation, Division of Mines and Geology, 1997, Guidelines for Evaluating and Mitigating Seismic Hazards in California, Special Publication 117, dated March 13.

California Department of Conservation, Division of Mines and Geology, 1998, Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada: International Conference of Building Officials, dated February.

California Department of Conservation, Division of Mines and Geology, State of California, 1999, Seismic Hazard Zones Official Map, Venice Quadrangle, 7.5-Minute Series: Scale 1:24,000, Open-File Report 98-27, dated March 25.

California Department of Transportation, 1991, NEWCON90, dated April 30.

California Department of Transportation (Caltrans), 2003, Corrosion Guidelines, Version 1.0, Division of Engineering Services, Materials Engineering and Testing Services, Corrosion Technology Branch, dated September.

California Department of Transportation, 2006, Highway Design Manual (updated September 1, 2006), http://www.dot.ca.gov/hq/oppd/pavement/guidance.htm, accessed September 14.

California Geological Survey (CGS), 2004, Checklist for the Review of Engineering Geology and Seismology Reports for California Public Schools, Hospitals, and Essential Services Buildings: Note 48, dated January 1.

Cao, T., Bryant, W. A., Rowshandel, B., Branum, D., and Willis, C. J., 2003, The Revised 2002 California Probabilistic Seismic Hazards Maps: California Geological Survey, dated June.

City of Los Angeles, 1996, Safety Element of the Los Angeles City General Plan, adopted November 26.

Hart, E.W., and Bryant, W.A., 1997, Fault-Rupture Hazard Zones in California, Alquist-Priolo Earthquake Fault Zoning Act with Index to Earthquake Fault Zone Maps: California Department of Conservation, Division of Mines and Geology, Special Publication 42, with Supplements 1 and 2 added in 1999.

Hartley, J.D., and Duncan, J.M., 1987, E' and Its Variation with Depth: American Society of Civil Engineers (ASCE), Journal of Transportation Engineering, Vol. 113, No. 5, dated September.

International Conference of Building Officials, 1997, Uniform Building Code, dated May 1.

Ishihara, K., 1995, Effects of At-Depth Liquefaction on Embedded Foundations During Earthquakes, Proceedings of the Tenth Asian Regional Conference on Soil Mechanics and Foundation Engineering, August 29 through September 2, Beijing, China, Vol. 2, pp. 16-25.

Jennings, C.W., 1994, Fault Activity Map of California and Adjacent Areas: California Division of Mines and Geology, California Geologic Data Map Series, Map No. 6, Scale 1:750,000.

Joint Cooperative Committee of the Southern California Chapter of the American Public Works Association and Southern California Districts of the Associated General Contractors of California, 2005, "Greenbook," Standard Specifications for Public Works Construction: BNI Building News, Los Angeles, California.

Kramer, S.L., 1996, Geotechnical Earthquake Engineering, Prentice Hall.

Ninyo & Moore, In-House Proprietary Information.

Ninyo & Moore, 2006, Limited Geotechnical Evaluation, NRG El Segundo Power Redevelopment, El Segundo, California, dated November 10.

Ninyo & Moore, 2007a, Proposal for Additional Geotechnical Services, NRG El Segundo Power Redevelopment Project, El Segundo, California, dated January 23.

Ninyo & Moore, 2007b, Supplemental Geotechnical Evaluation (Draft), NRG El Segundo Power Redevelopment, El Segundo, California, dated March 21.

Norris, R.M. and Webb, R.W., 1990, Geology of California: John Wiley & Sons, 541 pp.

Peterson, M.D., Bryant, W.A., Cramer, C.H., Cao, T., Reichle, M.S., Frankel, A.D., Lienkaemper, J.J., McCrory, P.A., and Schwartz, D.P., 1996, Probabilistic Seismic Hazard Assessment for the State of California: California Department of Conservation, Division of Mines and Geology Open File Report 96-08.

Seed, H.B., and Idriss, I.M., 1982, Ground Motions and Soil Liquefaction During Earthquakes, Volume 5 of Engineering Monographs on Earthquake Criteria, Structural Design, and Strong Motion Records: Berkeley, Earthquake Engineering Research Institute.

Tokimatsu, K., and Seed, H.B., 1987, Evaluation of Settlements in Sands Due to Earthquake Shaking, Journal of Geotechnical Engineering, ASCE, 113(8), 861-878.

United States Geological Survey, 1964 (Photorevised 1981), Venice, California Quadrangle Map, 7.5 Minute Series: Scale 1:24,000.

Youd, T.L., and Idriss, I.M. (Editors), 1997, Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils, Salt Lake City, Utah, January 5 through 6, 1996, NCEER Technical Report NCEER-97-0022, Buffalo, New York.

AERIAL PHOTOGRAPHS				
Source	**Scale**	**Date**	**Flight**	**Numbers**
USDA	1:20,000	11-19-53	AXJ-14K	73 and 74

REFERENCE: 2007 THOMAS GUIDE FOR LOS ANGELES/ORANGE COUNTIES, STREET GUIDE AND DIRECTORY

0 2400 4800

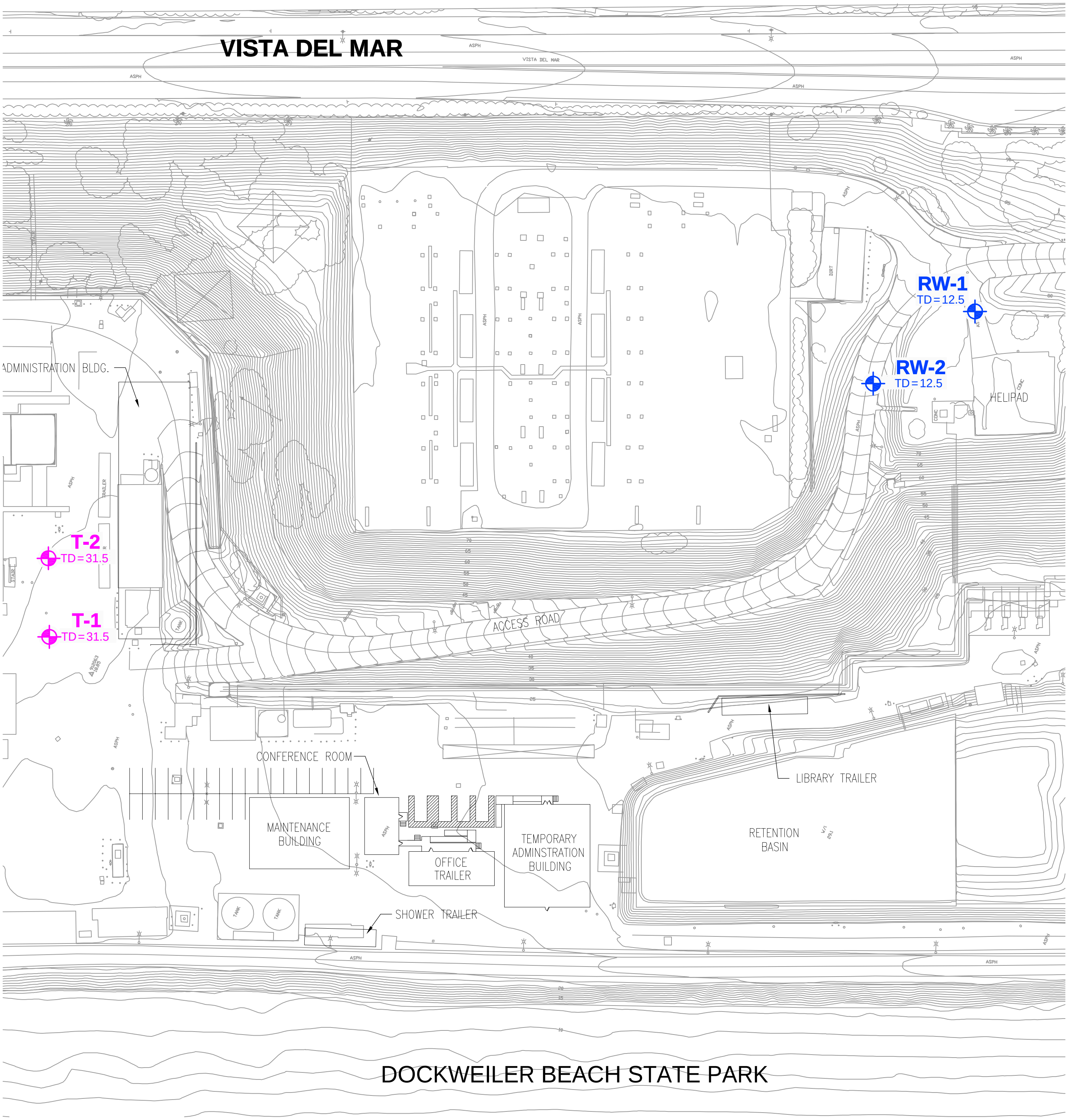
APPROXIMATE SCALE IN FEET

N

206954-A4.DWG

NOTE: ALL DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

Ninyo & Moore		SITE LOCATION MAP	FIGURE
PROJECT NO.	DATE	NRG EL SEGUNDO POWER REDEVELOPMENT EL SEGUNDO, CALIFORNIA	1
206954002	4/07		



N

APPROXIMATE SCALE IN FEET

0 100 200

LEGEND

T-1 TD=31.5	APPROXIMATE LOCATION OF EXPLORATORY BORING FOR CONDENSATE TANKS TD=TOTAL DEPTH IN FEET
RW-1 TD=12.5	APPROXIMATE LOCATION OF EXPLORATORY BORING FOR ROAD WIDENING TD=TOTAL DEPTH IN FEET

REFERENCE: BRINDERSON, 2006, NRG-EL SEGUNDO PROPOSED SITE PLAN, DATED OCTOBER 9.
NOTE: ALL DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

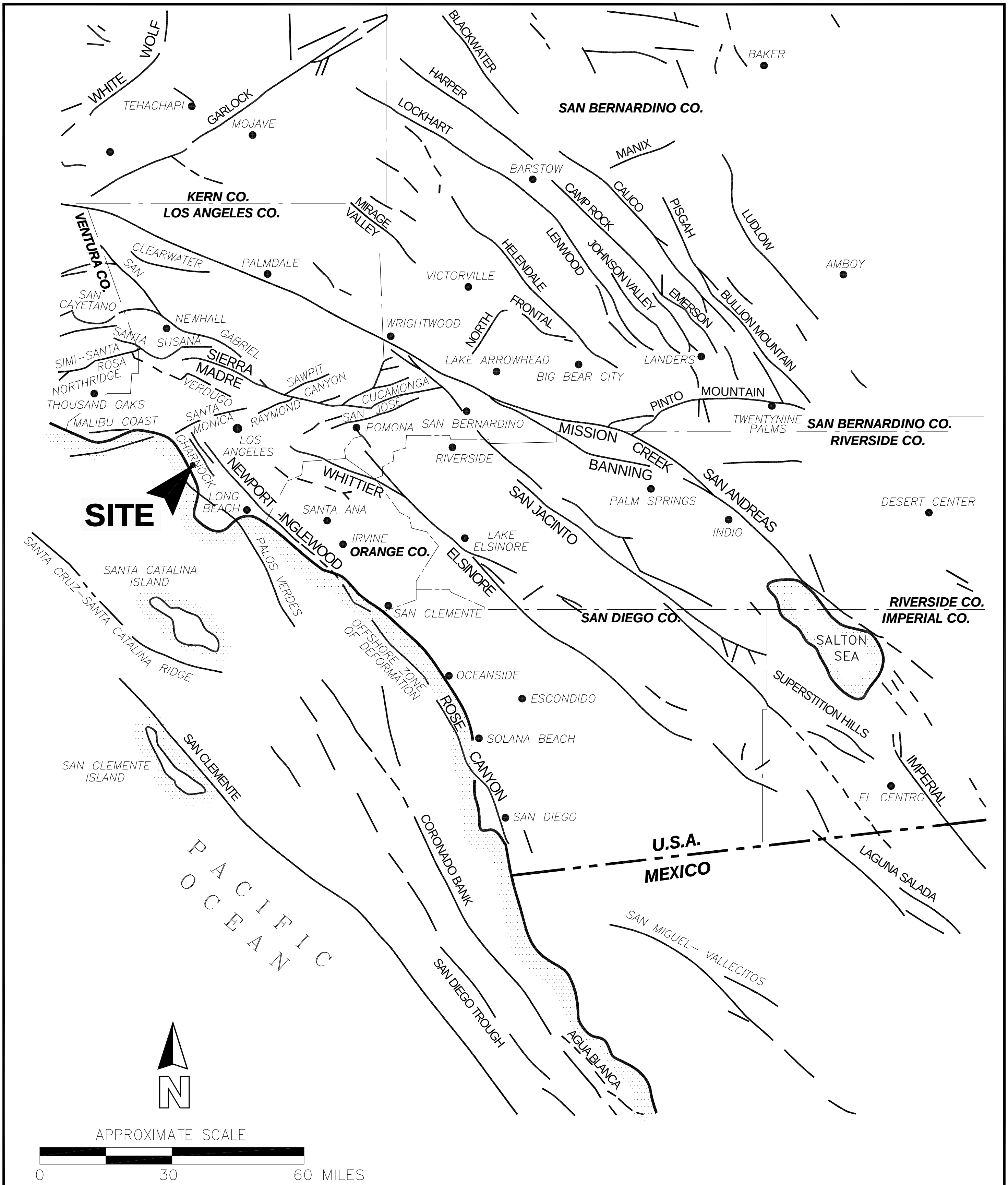
206954-A3.DWG

Ninyo & Moore

BORING LOCATION MAP

NRG EL SEGUNDO POWER REDEVELOPMENT
EL SEGUNDO, CALIFORNIA

PROJECT NO.	DATE	FIGURE
206954002	4/07	2



NOTE: ALL DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

206954-A5.DWG

Ninyo & Moore

FAULT LOCATION MAP

NRG EL SEGUNDO POWER REDEVELOPMENT
EL SEGUNDO, CALIFORNIA

PROJECT NO.	DATE	FIGURE
206954002	4/07	3

APPENDIX A

BORING LOGS

Field Procedure for the Collection of Disturbed Samples

Disturbed soil samples were obtained in the field using the following methods.

Bulk Samples

Bulk samples of representative earth materials were obtained from the exploratory borings. The samples were bagged and transported to the laboratory for testing.

The Standard Penetration Test (SPT) Sampler

Disturbed drive samples of earth materials were obtained by means of a Standard Penetration Test sampler. The sampler is composed of a split barrel with an external diameter of 2 inches and an unlined internal diameter of $1^{3}/_{8}$ inches. The sampler was driven into the ground 12 to 18 inches with a 140-pound hammer free-falling from a height of 30 inches in general accordance with ASTM D 1586-99. The blow counts were recorded for every 6 inches of penetration; the blow counts reported on the logs are those for the last 12 inches of penetration. Soil samples were observed and removed from the sampler, bagged, sealed and transported to the laboratory for testing.

Field Procedure for the Collection of Relatively Undisturbed Samples

Relatively undisturbed soil samples were obtained in the field using the following method.

The Modified Split-Barrel Drive Sampler

The sampler, with an external diameter of 3 inches, was lined with 1-inch-long, thin brass rings with inside diameters of approximately 2.4 inches. The sampler barrel was driven into the ground with the weight of a hammer or the kelly bar of the drill rig in general accordance with American Society for Testing and Materials (ASTM) D 3550-01. The driving weight was permitted to fall freely. The approximate length of the fall, the weight of the bar, and the number of blows per foot of driving are presented on the boring log as an index to the relative resistance of the materials sampled. The samples were removed from the sampler barrel in the brass rings, sealed, and transported to the laboratory for testing.

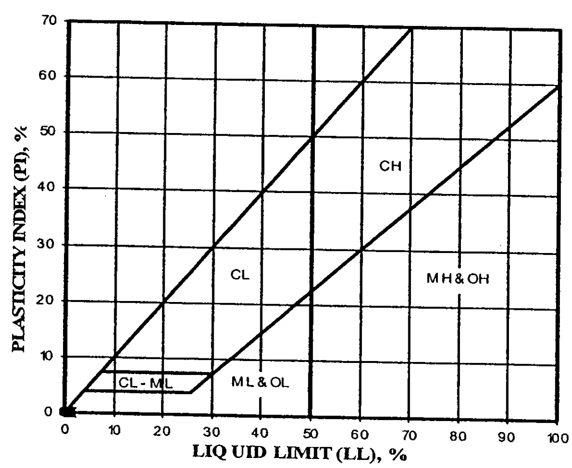
U.S.C.S. METHOD OF SOIL CLASSIFICATION

MAJOR DIVISIONS			SYMBOL	TYPICAL NAMES
COARSE-GRAINED SOILS	(More than 1/2 of soil >No. 200 sieve size)	GRAVELS (More than 1/2 of coarse fraction > No. 4 sieve size)	GW	Well graded gravels or gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines
			GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
		SANDS (More than 1/2 of coarse fraction <No. 4 sieve size)	SW	Well graded sands or gravelly sands, little or no fines
			SP	Poorly graded sands or gravelly sands, little or no fines
			SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS	(More than 1/2 of soil <No. 200 sieve size)	SILTS & CLAYS Liquid Limit <50	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean
			OL	Organic silts and organic silty clays of low plasticity
		SILTS & CLAYS Liquid Limit >50	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity, organic silty clays, organic silts
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils

GRAIN SIZE CHART

CLASSIFICATION	RANGE OF GRAIN SIZE	
	U.S. Standard Sieve Size	Grain Size in Millimeters
BOULDERS	Above 12"	Above 305
COBBLES	12" to 3"	305 to 76.2
GRAVEL Coarse Fine	3" to No. 4 3" to 3/4" 3/4" to No. 4	76.2 to 4.76 76.2 to 19.1 19.1 to 4.76
SAND Coarse Medium Fine	No. 4 to No. 200 No. 4 to No. 10 No. 10 to No. 40 No. 40 to No. 200	4.76 to 0.075 4.76 to 2.00 2.00 to 0.420 0.420 to 0.075
SILT & CLAY	Below No. 200	Below 0.075

PLASTICITY CHART



Ninyo & Moore

U.S.C.S. METHOD OF SOIL CLASSIFICATION

BORING LOG EXPLANATION SHEET

DEPTH (feet)	SAMPLES Bulk / Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	
0							Bulk sample.
							Modified split-barrel drive sampler.
							No recovery with modified split-barrel drive sampler.
							Sample retained by others.
							Standard Penetration Test (SPT).
5							No recovery with a SPT.
		XX/XX					Shelby tube sample. Distance pushed in inches/length of sample recovered in inches.
							No recovery with Shelby tube sampler.
							Continuous Push Sample.
							Seepage.
10							Groundwater encountered during drilling.
							Groundwater measured after drilling.
						SM	ALLUVIUM: Solid line denotes unit change.
							Dashed line denotes material change.
15							Attitudes: Strike/Dip b: Bedding c: Contact j: Joint f: Fracture F: Fault cs: Clay Seam s: Shear bss: Basal Slide Surface sf: Shear Fracture sz: Shear Zone sbs: Sheared Bedding Surface
20							The total depth line is a solid line that is drawn at the bottom of the boring.

Ninyo & Moore

BORING LOG

EXPLANATION OF BORING LOG SYMBOLS

PROJECT NO.	DATE	FIGURE
	Rev. 01/03	

DATE DRILLED 2/14/07 BORING NO. T-1

GROUND ELEVATION 19' ± (MLLW) SHEET 1 OF 2

METHOD OF DRILLING 8" Hollow-Stem Auger (Martini Drilling)

DRIVE WEIGHT 140 lbs. (Auto. Trip Hammer) DROP 30"

SAMPLED BY VAM LOGGED BY VAM REVIEWED BY SG

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								ASPHALT CONCRETE: Approximately 5 inches thick.
							GP	AGGREGATE BASE: Brown, moist, medium dense, sandy GRAVEL; approximately 9 inches thick.
							SP	OLDER ALLUVIUM: Brown, moist, medium dense, poorly graded SAND; orange oxidation staining.
5			32	5.5	103.9			
			17 (SSW)					
10			16					
			17 (SSW)					
				▽				@13.5': Groundwater encountered during drilling. Saturated.
			14 (SSW)					
15								Light gray; slight petroleum odor.
			35					
							SM	Light gray, medium dense, saturated, silty SAND.
20								

Ninyo & Moore

BORING LOG

NRG El Segundo Power Redevelopment
El Segundo, California

PROJECT NO.	DATE	FIGURE
206954002	4/07	A-1

DATE DRILLED 2/14/07 BORING NO. T-1

GROUND ELEVATION 19' ± (MLLW) SHEET 2 OF 2

METHOD OF DRILLING 8" Hollow-Stem Auger (Martini Drilling)

DRIVE WEIGHT 140 lbs. (Auto. Trip Hammer) DROP 30"

SAMPLED BY VAM LOGGED BY VAM REVIEWED BY SG

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
20		■	25				SP/SM	OLDER ALLUVIUM: (Continued) Alternating layers of dark gray, saturated, dense, poorly graded SAND and silty SAND; moderate petroleum odor.
								Gravel.
25		■	50/6"					Very dense; slight petroleum odor.
30		■	37					No gravel.
								Total Depth = 31.5 feet. Groundwater encountered during drilling at approximately 13.5 feet. Backfilled with bentonite slurry and capped with concrete on 2/14/07. Notes: Soil cuttings collected from below approximately 15 feet were placed in drums for profile testing and disposal by Shaw, Stone & Webster. Groundwater may rise to a level higher than that measured in borehole due to seasonal variations in precipitation and several other factors as discussed in the report. SSW = Samples collected by Shaw, Stone & Webster.
35								
40								

DATE DRILLED	2/14/07	BORING NO.	T-2
GROUND ELEVATION	19' ± (MLLW)	SHEET	1 OF 2
METHOD OF DRILLING	8" Hollow-Stem Auger (Martini Drilling)		
DRIVE WEIGHT	140 lbs. (Auto. Trip Hammer)	DROP	30"
SAMPLED BY	VAM	LOGGED BY	VAM
REVIEWED BY	SG		

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								ASPHALT CONCRETE: Approximately 4 1/2 inches thick.
							GP	AGGREGATE BASE: Brown, moist, medium dense, sandy GRAVEL; approximately 13 inches thick.
							SP	OLDER ALLUVIUM: Brown, moist, loose to medium dense, poorly graded SAND.
5			14	3.4	100.1			
			10 (SSW)					Medium dense.
10			13					Light brown.
			14 (SSW)					
				∇				@13.5': Groundwater encountered during drilling.
15			29					Dark gray; saturated; medium dense; strong petroleum odor; sample disposed of on-site.
			30 (SSW)					Dense.
20								

Ninyo & Moore

BORING LOG

NRG El Segundo Power Redevelopment
El Segundo, California

PROJECT NO.	DATE	FIGURE
206954002	4/07	A-3

DATE DRILLED	2/14/07	BORING NO.	T-2
GROUND ELEVATION	19' ± (MLLW)	SHEET 2 OF 2	
METHOD OF DRILLING	8" Hollow-Stem Auger (Martini Drilling)		
DRIVE WEIGHT	140 lbs. (Auto. Trip Hammer)	DROP	30"
SAMPLED BY	VAM	LOGGED BY VAM	REVIEWED BY SG

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
20		◣	40				SP	OLDER ALLUVIUM: (Continued) Dark gray, saturated, very dense, poorly graded SAND; strong petroleum odor; sample disposed of on-site.
							SM	Dark gray, saturated, dense, silty SAND; strong petroleum odor; sample disposed of on-site.
25		◣	30					
							SP	Dark gray, saturated, very dense, poorly graded SAND; strong petroleum odor; sample disposed of on-site.
30		◣	48					
35								Total Depth = 31.5 feet. Groundwater encountered during drilling at approximately 13.5 feet. Backfilled with bentonite slurry and capped with concrete on 2/14/07. Notes: Soil cuttings collected from below approximately 15 feet were placed in drums for profile testing and disposal by Shaw, Stone & Webster. Groundwater may rise to a level higher than that measured in borehole due to seasonal variations in precipitation and several other factors as discussed in the report. SSW = Samples collected by Shaw, Stone & Webster.
40								

Ninyo & Moore

BORING LOG

NRG El Segundo Power Redevelopment
El Segundo, California

PROJECT NO.	DATE	FIGURE
206954002	4/07	A-4

DATE DRILLED	2/14/07	BORING NO.	RW-1
GROUND ELEVATION	76' ± (MLLW)	SHEET	1 OF 1
METHOD OF DRILLING	6" Hollow-Stem Auger (Martini Drilling)		
DRIVE WEIGHT	140 lbs. (Auto. Trip Hammer)	DROP	30"
SAMPLED BY	VAM	LOGGED BY	VAM
REVIEWED BY	SG		

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								ASPHALT CONCRETE: Approximately 4 inches thick.
							GP	AGGREGATE BASE: Brown, dry, medium dense, sandy GRAVEL; approximately 10 inches thick.
	X						SP	EOLIAN DEPOSITS: Brown, dry, loose, poorly graded SAND.
	X			1.6				
5		X	6 (SSW)					Light brown; dry to damp.
10		X	9					Medium dense.
		X	15 (SSW)					
								Total Depth = 12.5 feet. No groundwater encountered during drilling. Backfilled with bentonite slurry and capped with concrete on 2/14/07.
15								Notes: Soil cuttings placed in drums for profile testing and disposal by Shaw, Stone & Webster. PID readings in breathing zone - 0 ppm. Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. SSW = Samples collected by Shaw, Stone & Webster.
20								

Ninyo & Moore

BORING LOG

NRG El Segundo Power Redevelopment
El Segundo, California

PROJECT NO.	DATE	FIGURE
206954002	4/07	A-5

DATE DRILLED 2/14/07	BORING NO. RW-2	
GROUND ELEVATION 68' ± (MLLW)	SHEET 1 OF 1	
METHOD OF DRILLING 6" Hollow-Stem Auger (Martini Drilling)		
DRIVE WEIGHT 140 lbs. (Auto. Trip Hammer)	DROP 30"	
SAMPLED BY VAM	LOGGED BY VAM	REVIEWED BY SG

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								ASPHALT CONCRETE: Approximately 6 inches thick.
							GP	AGGREGATE BASE: Light brown to brown, damp to moist, medium dense, sandy GRAVEL; approximately 8 inches thick.
							SP	FILL: Reddish brown, damp to moist, medium dense, poorly graded SAND.
				2.4				Light brown.
5			11 (SSW)					
								Brown; asphalt fragment.
							SP	EOLIAN DEPOSITS: Light brown, damp to moist, medium dense, poorly graded SAND.
10			14					
			16 (SSW)					
								Total Depth = 12.5 feet. No groundwater encountered during drilling. Backfilled with bentonite slurry and capped with concrete on 2/14/07.
								Notes: Soil cuttings placed in drums for profile testing and disposal by Shaw Stone & Webster.
15								PID readings in breathing zone - 0 ppm.
								Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.
								SSW = Samples collected by Shaw, Stone & Webster.
20								

Ninyo & Moore

BORING LOG

NRG El Segundo Power Redevelopment
El Segundo, California

PROJECT NO.	DATE	FIGURE
206954002	4/07	A-6

APPENDIX B

LABORATORY TESTING

Classification

Soil materials were visually and texturally classified in accordance with the Unified Soil Classification System (USCS) in general accordance with ASTM D 2488-00. Soil classifications are indicated on the logs of exploratory borings in Appendix A.

In-Place Moisture and Density Tests

The moisture content and dry density of relatively undisturbed soil samples obtained from the exploratory borings were evaluated in general accordance with ASTM D 2937-04. The test results are presented on the logs of exploratory borings in Appendix A.

200 Wash

An evaluation of the percentage of particles finer than the No. 200 sieve was performed on selected soil samples in general accordance with ASTM D 1140-00. The results of the tests are presented on Figure B-1.

Direct Shear Tests

Direct shear tests were performed on relatively undisturbed soil samples in general accordance with ASTM D 3080-04 to evaluate the shear strength characteristics of selected earth materials. The samples were inundated during shearing to represent adverse field conditions. The test results are presented on Figures B-2 and B-3.

R-Value

The resistance value, or R-value, of near-surface site soils was evaluated in general accordance with California Test (CT) 301. Samples were prepared and evaluated for exudation pressure and expansion pressure. The equilibrium R-value is reported as the lesser or more conservative of the two calculated results. The test results are shown on Figure B-4.

Sand Equivalent

Sand equivalent (SE) test was performed on a selected representative sample in general accordance with ASTM D 2419-02. The SE value reported on Figure B-5 is the ratio of the coarse- to fine-grained particles in the selected sample.

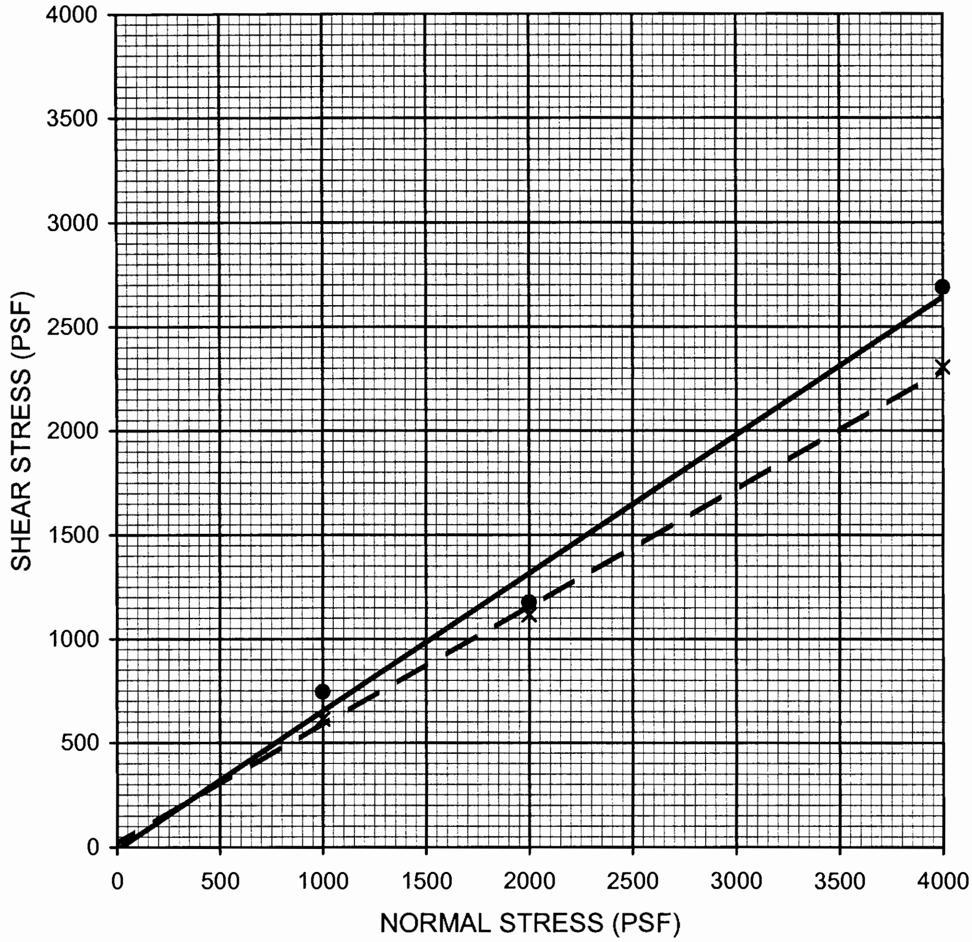
Soil Corrosivity Tests

Soil pH and resistivity tests were performed on representative samples in general accordance with CT 643. The sulfate and chloride content of selected samples were evaluated in general accordance with CT 417 and 422, respectively. The test results are presented on Figure B-6.

SAMPLE LOCATION	SAMPLE DEPTH (FT)	DESCRIPTION	PERCENT PASSING NO. 4	PERCENT PASSING NO. 200	USCS (TOTAL SAMPLE)
T-1	1.5-5.0	Poorly Graded SAND	99	4	SP
T-2	10.0-11.5	Poorly Graded SAND	99	1	SP
RW-1	10.0-11.5	Poorly Graded SAND	100	2	SP
RW-2	10.0-11.5	Poorly Graded SAND	100	1	SP

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 1140-00

Ninyo & Moore		NO. 200 SIEVE ANALYSIS	FIGURE
PROJECT	DATE	NRG El Segundo Power Redevelopment El Segundo, California	B-1
206954002	4/07		



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion, c (psf)	Friction Angle, ϕ (degrees)	Soil Type
Poorly Graded SAND	—●—	T-1	5.0-6.5	Peak	0	34	SP
Poorly Graded SAND	- - x - -	T-1	5.0-6.5	Ultimate	18	30	SP

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080-04

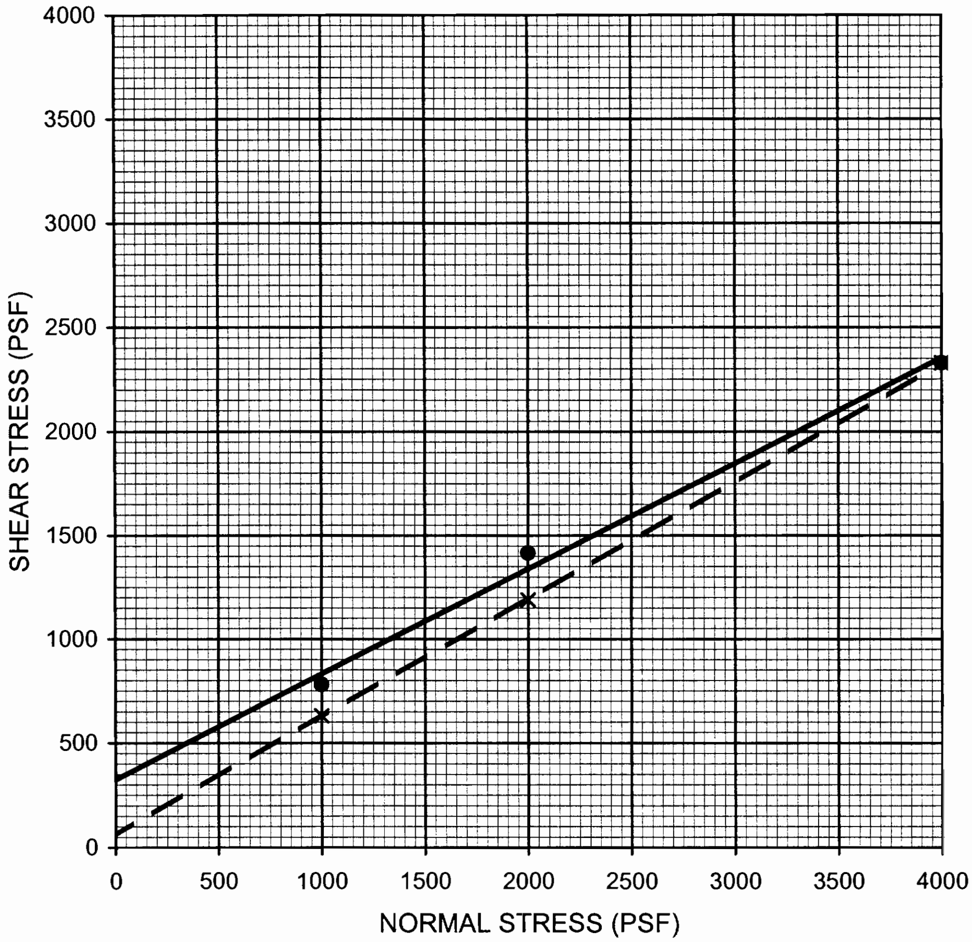
Ninyo & Moore

DIRECT SHEAR TEST RESULTS

NRG El Segundo Power Redevelopment
El Segundo, California

PROJECT	DATE
206954002	4/07

FIGURE
B-2



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion, c (psf)	Friction Angle, ϕ (degrees)	Soil Type
Poorly Graded SAND	—●—	T-2	5.0-6.5	Peak	324	27	SP
Poorly Graded SAND	- - x - -	T-2	5.0-6.5	Ultimate	60	30	SP

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080-04

Ninyo & Moore		DIRECT SHEAR TEST RESULTS	FIGURE
PROJECT	DATE	NRG El Segundo Power Redevelopment El Segundo, California	B-3
206954002	4/07		

SAMPLE LOCATION	SAMPLE DEPTH (FT)	SOIL TYPE	R-VALUE
RW-1	1.2-5.0	SP	73

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2844-01

Ninyo & Moore

PROJECT	DATE
206954002	4/07

R-VALUE TEST RESULTS

NRG El Segundo Power Redevelopment
El Segundo, California

FIGURE
B-4

SAMPLE LOCATION	SAMPLE DEPTH (FT)	SOIL TYPE	SAND EQUIVALENT
T-1	1.5-5.0	SP	95

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2419-02

Ninyo & Moore		SAND EQUIVALENT VALUE	FIGURE
PROJECT NO.	DATE	NRG El Segundo Power Redevelopment El Segundo, California	B-5
206954002	4/07		

206954002 B-5 Sand, T-1 -- 1.5- 5.0.xls

SAMPLE LOCATION	SAMPLE DEPTH (FT)	pH [1]	RESISTIVITY [1] (Ohm-cm)	SULFATE CONTENT [2]		CHLORIDE CONTENT [3] (ppm)
				(ppm)	(%)	
T-1	1.5-5.0	7.5	6,700	90	0.009	115
T-2	1.5-5.0	7.5	6,097	125	0.013	90

[1] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 643
[2] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 417
[3] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 422

Ninyo & Moore		CORROSIVITY TEST RESULTS	FIGURE
PROJECT	DATE	NRG El Segundo Power Redevelopment El Segundo, California	B-6
206954002	4/07		